August 23, 2020

Submitted via electronic mail:
rickey.d.james.civ@mail.mil

The Honorable Rickey D. James
Assistant Secretary of Army, Civil Works
101 Army Pentagon, Room 3E700
Washington, DC 20310–0101

Re: Final Environmental Impact Statement, No. 20200148
Pebble Mine, AL

Dear Assistant Secretary James,

Thank you for the opportunity to comment on the Army Corps of Engineers ("Corps") final environmental impact statement ("FEIS" or "Report") for the Pebble mine proposed by the Pebble Limited Partnership ("PLP") in the Bristol Bay area of Alaska. The PLP is 100% owned by The Northern Dynasty Partnership, which is a wholly owned Canadian-based subsidiary of Northern Dynasty Minerals, Limited.

First, I appreciate the hard work of the Corps staff in preparing the FEIS and the thousands of pages and hours involved in that lengthy review. However, after reviewing the proposed mine plan and the environmental impacts, I have major lingering concerns regarding the adequacy of the Corps' environmental review and the FEIS.

Based on my review of the proposed Pebble project and the FEIS documents, I have grown convinced that the Pebble mine cannot under any circumstances be developed as proposed without significant, long-lasting and unavoidable adverse environmental impacts. I am persuaded that no option under consideration by Pebble Limited Partnership ("PLP") for the mine would result in a safe and sustainable mine. Moreover, no amount of compensatory mitigation required to replace the thousands of acres of destroyed wetlands and stream miles can change that fact.

As the CEO of Sabin Metal Corporation, the largest independently owned precious metal refiner in North America, I understand the mining business very well. I am pro-mining, pro-growth and strongly support this Administration's focus on creating jobs. That alone might seem like a good reason to support the Pebble Mine project, but when the costs and benefits are weighed, the benefits simply do not add up and the risks of moving forward are too grave.

Background

The proposed Pebble mine sits along the headwaters of the Nushagak and Kvichak rivers. These two rivers produce half of Bristol Bay's sockeye salmon and the Bay produces half of the world's wild pacific sockeye salmon. The Nushagak is also the world's most prolific king salmon river. Up to 60 million salmon return home every spring to spawn in these wild and pristine rivers. Simply put, the area is one of a kind and is simply irreplaceable.

This is a very bad project for many reasons, least of which the project will have widespread impacts on sensitive area from construction alone, resulting in the permanent loss of at least 2,226 acres of wetlands and other waters, including 104 miles of streams.

Moreover, there are genuine questions regarding the financial viability of the project. Four of the world's major mining companies, Cominco, Rio Tinto, Anglo American, and Mitsubishi previously sought to develop the mine but ultimately abandoned the idea, and walked away from hundreds of millions of dollars in sunk costs because they concluded there is simply no safe way to develop it. The difficulty of PLP in attracting the needed capital, coupled with the financial challenges of the PLP's parent company, raises serious doubts about the Pebble's financial viability long-term, including the ability to provide adequate financial assurances to properly operate and close the mine. Given China's continued aggressive acquisition of mineral assets around the globe, including state-owned companies' focus on acquiring Canadian mining companies,[1] it would come as no surprise that, if and when Pebble permits are granted, that Pebble may eventually be owned by the Chinese.

While there are many reasons that the Pebble mine should not be allowed to go forward, my focus is on lack of adequate consideration of the water management system needed in perpetuity to ensure the safety of the mine and the potential catastrophic failure of the massive tailings dam, an event that would forever destroy an irreplaceable fishery and the many thousands of jobs dependent upon those resources.

As discussed in detail below, given the location and extraordinary risks from mining in the Bristol Bay area, there is simply no way to proceed safely with Pebble mine, and to proceed would not only be in violation NEPA, but would be morally irresponsible and reckless.

<u>The FEIS Violates NEPA by Failing to Adequately Review the Adverse Impacts from a Catastrophic Failure of the Tailings Dam; These Concerns Alone Are a Sufficient Basis to Deny Future Permits</u>

If constructed, the Pebble mine tailings storage facility ("TSF") would be one of the world's largest mine waste repositories and dam embankments. Unlike a dam designed and built to impound water that can be drained if the dam loses structural integrity, tailings embankments for holding mine wastes must be built to function forever.

Although NEPA does not mandate any particular outcome for a major federal action, such as the permitting of Pebble mine, that will significantly impact the environment, NEPA requires an agency to take a "hard look" at the environmental consequences of the project and the government's action. *Northwest Envtl. Advocates v. Nat'l Marine Fisheries Serv.*, 460 F.3d

[1] The Chinese recently acquired the Canadian mining company, Nunavut Gold. https://www.cbc.ca/news/canada/north/tmac-resources-purchase-agreement-1.5576240#:~:text=China's%20Shandong%20Gold%20Mining%20has,Hope%20Bay%20mine%20in%20Nunavut.

1125 (9th Cir. 2006). NEPA also requires an agency to analyze "reasonably foreseeable significant adverse effects," including "impacts which have catastrophic consequences, even if their probability of occurrence is low, provided that the analysis of the impacts of supported by scientific evidence, is not based on pure conjecture, and is within the rule of reason." 40 C.F.R. § 1502.22. The prospect of a failure of the Pebble Mine TSF fits within these requirements, which were not modified by the 2020 amendments to the Council on Environmental Quality's NEPA regulations. A "hard look" also requires an agency to review any new information provided during the review period. *Sierra Club v. Bosworth*, 2005 U.S. Dist. LEXIS 27573 (N.D.Ca., November 14, 2005).

Recently, the Corps' experts, AECOM, raised serious concerns regarding the stability of the dam proposed by PLP, yet the potential for such a dam failure, along with the likely impacts, was not even discussed in the FEIS.[2] (AECOM report attached) PLP's design of the TSF is only conceptual and, as noted by AECOM, its purported safety is based on information in literature, because testing completed to date on the bulk tailings has been minimal. PLP's response to the Corps' concerns regarding the TSF, in reliance upon the literature, is according to AECOM "incomplete and misleading," based PLP's on flawed assumptions regarding the percent of solids composing the tailings and their capacity for segregation. As the FEIS concludes, "[t]herefore, there is much uncertainty in evaluating the stability of the mine site embankments based on a conceptual-level design." This magnitude of uncertainty is alarming and unacceptable at best, given what is at stake if a dam failure were to occur. Given the lack of information and detail regarding the final design, the Corps is obligated to conduct a worst-case scenario analysis that would analyze the scope of impacts from a catastrophic failure of the dam. *Sierra Club v. Sigler,* 695 F.2d 957, 972 (5th Cir. 1983) (holding NEPA requires a worst case scenario was required for a low probability/catastrophic event).

Tailings dam failures are not infrequent. Over the past five decades, 63 major tailings dam failures have been reported worldwide.[3] And since 1990 the frequency of high-consequence failures has increased to 5-6 significant tailings dam failures annually. The primary cause of these dam failures often relates directly to either seismic activity or meteorological events, such a significant snow or rainfall event, or a combination of both.[4] Not only is effective

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[2] AECOM's technical memo states:

> *We remain concerned that there are uncertainties as to whether the 55 percent thickened tailings planned by PLP would segregate enough to promote reduction of the phreatic surface near the embankment, which translates to uncertainties regarding the effect of the tailings segregation on embankment stability.*

Id. at 2.

[3] Owen, J.R. et al. 2020. *Catastrophic Tailing Dam Failures and Disaster Risk Disclosure,* Intl. Jnl. Of Disaster Risk Reduction, Vol. 42.

[4] Zongjie, L, et al. 2019. *A Comprehensive Review on Reasons for Tailings Dam Failures Based on Case History*, Advances in Civil Engineering, Vol. 2019

Rico, M., et al. 2008. *Reported tailing dam failures: A review of the European incidents in the worldwide context. Journal of Hazardous Materials*. 152(2):846–852.

water management a critical element to ensuring long-term stability of the tailings dam, the chemical and physical composition of the tailings and how they are disposed is central to such safety.

Importantly, saturation of part, or all, of a tailings dam can lead to what is referred to as a static load-induced liquefaction, which involves the loss of strength in saturated material because of the build-up of pore water pressures. As AECOM notes in its technical memo, the age of the tailing deposits is also likely to affect liquefaction, with newer depots and loose fills without compaction being for more susceptible to liquefaction and seismic hazards.[5]

The Corps' regulations for the conduct of its "public interest review" of permit applications explicitly call for an examination of the safety of impoundment structures. 33 C.F.R. § 320.4(a)(1)(k). It does not appear that the Corps has followed this regulation in its review of the Pebble Mine. The near collapse of the Oroville Dam in California in 2017 shows some of the dangers in failure to subject dam safety to rigorous NEPA review.

There have been many who have criticized PLP's plan for water management, including myself, and some experts who believe that water management is the Achilles heel of the project. It is simply not possible for the water management system proposed by PLP to work effectively long-term, given the unprecedented scale (sheer amount of water) and the need to have a very complex treatment system operate perfectly, all the time, and for centuries into the future. At some point, contaminated wastewater will be released into the environment and impact the local streams and fisheries. The extent of such impact remains unknown and the FEIS fails to address this aspect of the mine's water management and the concerns being raised by those rightly concerned about the viability of the mine and protection of the resources.

I raise these concerns, because Pebble mine is located in an area that is known to be seismically active and even AECOM predicts that "moderate to large earthquakes . . . can be expected to occur during the life of the mine."[6] Although the FEIS acknowledges the risks in terms of "unstable slopes" and "slope failure" potentially caused by earthquakes and an increase in precipitation and freeze-thaw events (that have been occurring in the region due to climate change), the FEIS fails to specifically discuss a catastrophic dam failure analysis and assess the scope of impacts to the downstream environment, including the fisheries, if such an event were to occur.[7]

At a minimum, the Corps must evaluate the adverse impact on fisheries if such a catastrophic failure were to occur. How far downstream would tailings be released? What

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[5] AECOM at 3.
[6] FEIS at 4.15-16
[7] FEIS at 4.15-26; 3.15-12.

> *Slope failure can also be triggered by earthquakes. Increased precipitation due to climate change is predicted to occur as rain and snow in the Iliamna Lake and Cook Inlet areas over the next few decades (SNAP 2019), which could locally increase the risk of landslides and avalanches.*

would be the impact on water quality and streams critical for the salmon reproduction? What would be the economic impacts on the region? What would be the cost of the cleanup and would such a cleanup even be feasible? These are answerable questions that should be answered before the project is allowed to proceed.

The risks of a catastrophic dam failure are more than just speculative or hypothetical, as Alaska experiences earthquakes on a magnitude of 6-7 at least five times each year. And the Bristol Bay area experiences more than 500 earthquakes each year, and although many are minor, in January of this year, the area experienced an earthquake measuring 3.6 magnitude, larger than the earthquake causing the 2015 Fundao dam collapse in Brazil.[8]

AECOM's technical review looked at a number of mine dam collapses, including the Fundao dam, which impacted an area of over 5,400 acres, including 415 miles of waterways. The Fundao dam collapse was caused by a minor earthquake of only 2.7 in magnitude. At the time of its collapse, the Fundao open pit was estimated to contain approximately 57 million cubic yards of tailings at a depth of 100 feet. In comparison, if allowed to move forward, Pebble Mine would eventually hold 2.5 billion cubic yards of tailings at depths of up to 600 feet deep.

AECOM's technical report was issued after the draft EIS and raises serious concerns and uncertainties involving assumptions and geologic conditions that could lead to the potential collapse of the tailings dam.[9] Despite the technical report raising these serious issues, the text of the FEIS itself glosses over the issue and fails to discuss it in any meaningful manner. As a result, the FEIS analysis on this issue, in particular, is incomplete and misleading. *NRDC v. United States Forest Serv.*, 421 F.3d 797, 808 (9th Cir. 2005) (holding that agency's "misleading" economic methodology violated NEPA's "procedural requirement to present complete and accurate information to decision makers and to the public to allow an informed comparison of the alternatives"). Although NEPA does not obligate an agency to engage in meaningless speculation, the potential failure of the Pebble Mine tailings dam is more than speculative given the mine's location, the geologic conditions, seismic activity in the area, and

[8] *See* https://www.nationalfisherman.com/viewpoints/alaska/ringof-fire-lights-up-earthquakes-near-proposed-pebble-mine-site
[9] The following are excerpts from the AECOM report:

- We are concerned that the analysis does not allow for a deeper failure surface to occur up to the depth of the lowest centerline raise." Pg. 4.
- Because several assumptions of the above analysis may be optimistic, the calculated FoSs are generally considered to be results based on effectively best-case or normal operating conditions, indicating that some potentially high-risk situations have not been evaluated." Pg. 4.
- There is concern that some and perhaps all of the entire centerline part of the bulk TSF main embankment (not just the uppermost raise) could slide into potentially undrained tailings and have consequent effects in a downstream direction. Pg. 8.
- A concern is that deep tailings could suddenly liquefy under static or dynamic (earthquake) loading, causing a containment failure and release that cannot be practically mitigated in a timely way. RFI 008q. Item #2: *Tailings Liquefaction and Seismic Stability.*

the Corps' own technical expert's stated concerns. Consequently, the dam's potential structural failure is a foreseeable event for which the consequences must be evaluated.

The Corps Violated NEPA by Failing to Obtain Comment from the USGS on Potential Impacts of an Earthquake

NEPA requires federal agencies to officially consult with other agencies possessing certain expertise, such as seismology, before making a final decision. Given the significant seismic activity in the Bristol Bay area, and the likely devastating consequences from such an event, the Corps should have officially solicited and obtained official comments from the agency with the most expertise, i.e., the USGS. The FEIS discusses generally the issue of seismic and geologic hazards on the TSF,[10] but since the PLP's plans are conceptual, relies upon future planning and design considerations by PLP without drawing any conclusions as to the overall safety of the tailings dam. Rather than rendering an opinion on the viability of a tailings dam, the FEIS merely offers recommendations for future analysis to be considered as design progresses for approval under State of Alaska's dam safety permitting.

Furthermore, there is not a single mention in the FEIS, let alone an informed discussion, on the potential impacts to fisheries. These deficiency leaves a gaping hole in understanding one of the most consequential risks and impacts of the Pebble mine. To date, while the Corps has merely relied upon USGS literature data for its own analysis and has not obtained official written comments from the USGS on the potential for a catastrophic failure of the tailings dam. Such failure has been deemed by some courts as a violation of NEPA.[11] *Warm Springs Dam Task Force v. Gribble*, 621 F.2d 1017, 1022 (9th Cir. 1980) (holding the Corps violated NEPA by failing to consult the USGS on potential catastrophic failure of dam due to earthquake activity in the area). *See also Save the Niobrara River Association v. Andrus* (483 F. Supp. 844 (D. Neb. 1979) (environmental impact statement for dam project held to be inadequate because, among other deficiencies, it failed to adequately examine geological instability).

Conclusion

Given the recent concerns raised by AECOM, the uncertainty related to the design and viability of the tailings dam, and the failure of the FEIS to meaningfully analyze and discuss the worst-case scenario involving a catastrophic dam failure, the FEIS is deficient and non-compliant with NEPA.

Given the high stakes involved, this level of uncertainty and the risks alone should warrant the denial of any future permits.

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[10] FEIS 4.15, Geohazards and Seismic Conditions.
[11] According to the FEIS, the probabilistic and deterministic seismic hazard analyses would be updated in final design, incorporating best practices for analysis and updated US Geological Survey.

Once, again, thank you for the opportunity to submit the foregoing comments.

Sincerely,

Andrew Sabin

Cc: President Donald Trump
Andrew Wheeler, Administrator, EPA
Dave Ross, Office of Water, EPA
Matt Leopold, General Counsel, EPA

Date: **December 13, 2019**

To: **Bill Craig, AECOM**

From: **Chuck Vita, PhD, PE, GE; Cecil Urlich, PE; and Nancy Darigo, PG, CEG; AECOM**

Subject: **Pebble Project EIS – Bulk TSF Embankment Seismic Stability Analysis**

1.0 INTRODUCTION

This technical memorandum provides a review of responses to issues raised in Requests for Information (RFIs) 008g and 008h, "Followup to Seismic Stability Analysis," regarding tailings liquefaction potential and stability of the centerline portion of the bulk tailings storage facility (TSF) main embankment. The responses to these RFIs were provided in Pebble Limited Partnership (PLP) 2019-RFI 008g, Knight Piésold (KP) (2019), and PLP 2019-RFI 008h. The response to RFI 008h is provided in Attachment A to this memorandum for reference.

The purpose of this memorandum is to inform the impacts analysis in the EIS regarding potential effects on the stability of the bulk TSF main embankment in the event of tailings liquefaction or high phreatic surface in the embankment, to identify uncertainties in the impact analysis, and to provide recommendations for future analysis to be considered as design progresses for approval under State of Alaska dam safety permitting.

2.0 REVIEW OF RFI 008H

2.1 TAILINGS SEGREGATION

The responses to Items #5a and 5b in RFI 008h discuss the basis for the assumption that tailings would segregate by grain size, from coarse tailings near the spigots at the bulk TSF main embankment crest to finer grain sizes in the middle of the bulk TSF impoundment. The current conceptual design relies on information in the literature (Vick 1990), because testing completed to date on the bulk tailings has been minimal.

The description of sedimentation processes in the RFI response that occur with conventional tailings deposition is usually true for slurry tailings, but not necessarily for thickened tailings, which do not segregate like slurry tailings do. MEND (2017) indicates that "Thickened tailings may be non-segregating, producing a tailings product with potentially low hydraulic conductivity and high moisture retention capacity.... Consistency will depend on the variability of the tailings properties." Thus, the summary of expected particle size sorting behavior based on Vick (1990) in the RFI response is incomplete and misleading. This is because Vick also states that:

> On the basis of copper tailings deposits, however, Volpe (1979) suggests that the overall variation in average permeability with distance is not very significant, only about a factor of 10, for tailings discharged at pulp densities of 45-50%. Data presented by Soderberg and Busch (1977) show even less systematic variation for some deposits, which exhibit almost random variations in permeability with distance.

PLP plans to pump thickened tailings at about 55 percent solids to the bulk TSF, which could result in even less segregation than that described by Volpe's 45-50 percent findings. The ability to operate as a flow-through drained facility can only be confirmed with Pebble-specific tailings testing, while also recognizing that "It could take months to years to optimize the thickening system to produce a consistent tailings product and the achieved solids content is often at least 5 percent lower than the design target" (MEND 2017).

As noted in Item #5c of the response to RFI 008h, several types of tailings testing would be completed during the preliminary design phase of the Alaska Dam Safety Program (ADSP) (ADNR 2017), which would occur after the EIS process is completed. The ADSP guidelines do not describe the testing procedures, and it is the responsibility of the dam designer to complete such testing in accordance with sound geotechnical engineering practices and current industry standards as part of the design needed to provide a safe and stable bulk TSF main embankment.

Having coarse tailings near the bulk TSF embankment crest is important for reducing the phreatic surface level as it approaches the embankment, which has implications for dam stability. As discussed in Section 2.3, one of the contributing factors of the recent failure of the Fundão Dam at Samarco Mine in Brazil were operational problems with achieving the planned tailings deposition and drainage objectives in silty versus sandy tailings at this flow-through mine (Morgenstern et al. 2016). We remain concerned that there are uncertainties as to whether the 55 percent thickened tailings planned by PLP would segregate enough to promote reduction of the phreatic surface near the embankment, which translates to uncertainties regarding the effect of tailings segregation on embankment stability.

2.2 Emergency Action Plan

The response to Item #5d of RFI 008h indicates that an Emergency Action Plan (EAP) would include procedures to be implemented in the event that TSF water levels were found to exceed defined maximum operating levels. It is unclear if such procedures would be included as a non-failure emergency condition in the EAP required under the ADSP (ADNR 2017).

Regardless, such EAP planning and procedures should be completed and ready for mitigation before the start of the bulk TSF construction and operations, so that they can be immediately implemented if and when needed. This means that the required work force, equipment and materials be readily available to immediately implement the required mitigations.

2.3 Depth of Liquefaction in Tailings

The response to Item #7 of RFI 008h describes the basis of the assumption for limiting the depth of liquefaction to 100 feet in the preliminary tailings liquefaction analysis (KP 2019). The response is based on two references, one of which (Kramer 1996) does not discuss liquefaction depths, and the other (Geo-Slope 2018) which does not address liquefaction at all.

Based on a review of the cited references, the depth assumption appears to be based on limited and selective interpretation of information on the relationship between void ratio, unit weight, and porewater pressures below 100 feet. Excess pore pressures less than that required for liquefaction (i.e., c < 1.00 but > 0) can be generated at depths greater than 100 feet, and any excess pore pressures would decrease the effective strength of affected tailings or embankment materials. Any reduction in effective strength would reduce embankment stability.

Appendix K3.15 of the EIS cites additional literature on the need to evaluate liquefaction potential deeper than 100 feet and possibly up to 1,000 feet. The limited depth of liquefaction assumption also contradicts the findings of recent TSF failure analyses, for example, the static liquefaction failure of the Fundão Dam at the Samarco Mine (Morgenstern et al. 2016; Reid 2019), and advice given to tailings dam designers by some of the world's experts in the analysis of liquefaction potential in mine tailings (Robertson 2010; Sadrekarimi 2014; Jefferies and Been 2016; Marr 2019).

Kramer (1996) states "The susceptibility of older soil deposits to liquefaction is generally lower than that of new deposits" and adds "Loose fills, such as those placed without compaction, are very likely to be susceptible to liquefaction. The stability of hydraulic fill dams and mine tailings piles, in which soil particles are loosely deposited by settling through water, remain an important contemporary seismic hazard." Therefore, special analyses are needed for new geologic soils like tailings. Marr (2019) warns that "text book classifications of sands as "drained" for stability can be terribly misleading."

The Fundão Dam failure investigation (Morgenstern et al. 2016) and follow-up analyses (Reid 2019) point to sandy tailings undergoing static liquefaction when silty tailings started to deform and release the confining stresses on the sandy tailings. The Fundão TSF had been having operational problems in achieving the planned tailings deposition and flow-through drainage objectives that resulted in several design changes being made as mine operations continued. Morgenstern et al. (2016) state "But the concept also had certain vulnerabilities. The design was not adaptable to variation in the proportion of sands and slimes received. And most importantly, it depended on achieving adequate drainage of the sands." Reid (2019) provides a slightly different view of the factors that contributed to the static liquefaction at Fundão. The sandy tailings zone that liquefied was estimated to be around 100 feet deep, but Reid's analyses indicate the zone could have been deeper.

2.4 Embankment Stability if Tailings Liquefy

Regardless of the basis of the liquefaction depth assumption and analyses described above, it is acknowledged that additional stability assessments assuming full depth of liquefaction in the tailings near the bulk TSF main embankment was provided under Items #7b and 7c of RFI 008h for failure surfaces in both the upstream and downstream directions (Figures 2 and 4, respectively). However, results were provided for a post-liquefaction case only (i.e., affected tailings at post-liquefaction residual strength immediately after an earthquake), but not for a case during the period of strong ground shaking that causes increased pore pressures that could lead to liquefaction.

Our central concern and issue is the overall bulk TSF main embankment stability. The critical combination of excess pore pressures and ground shaking resulting in minimum embankment stability may occur at any time during the period of ground shaking that could be caused by earthquake events, equipment activities, or blasting vibrations. Therefore, embankment stability must be analyzed based on both pore pressure buildup (strength loss) in the tailings *and* the embankment material, with continued strong ground motion during the full duration of ground shaking (as represented in design earthquake time history input motions).

As further discussed in Section 2.7, potential critical pore pressures in the main embankment must not be assumed away by claiming either only a favorable (very low or deep) phreatic surface in the main embankment, or such highly permeable embankment material that excess pore pressures (above hydrostatic) could not develop during design earthquake loading. A relatively high phreatic surface in the downslope part of the embankment in combination with high excess pore pressures during ground motions are credible adverse conditions that must be considered as part of a technically defensible analysis of long-term embankment stability.

Considering that the upstream part of the centerline raises are planned to be built partly over tailings, it is unclear if the post-liquefaction shallow failure surface shown in RFI 008h Figures 2 and 3 is constrained by the analysis to include only the uppermost centerline raise based on an assumption that the tailings would buttress any failure surface deeper than the top raise. We are concerned that the analysis does not allow for a deeper failure surface to occur up to the depth of the lowest centerline raise. Because the response to Item #7d regarding risk reduction in the event of tailings liquefaction only pertains to the upstream case, we are also concerned that more of the centerline part of the embankment below just the most recent raise could slide into potentially undrained tailings, setting the mass in motion with adverse consequential effects on the TSF in a downstream direction.

The fully liquefied, post-liquefaction case in a downstream direction shown in RFI 008h Figure 4 is based on an assumption of a deep phreatic surface with no excess pore pressures in the TSF main embankment (i.e., ru = 0). While a scenario with a higher phreatic surface is analyzed under Item #9c of RFI 008h (Section 2.7), the yield acceleration (ky) values in Table 3 can be considered practical upper bounds only (e.g., arguably too high, not conservative enough), and the reported factor of safety (FoS) and ky/peak ground acceleration (PGA) values in Table 3 considered conditional calculated values only, subject to the validity of the assumptions.

Because several assumptions in the above analyses may be optimistic, the calculated FoSs are generally considered to be results based on effectively best-case or normal operating conditions, indicating that some potentially high-risk situations have not been evaluated. Additional discussion of the high phreatic case is provided under Section 2.7. Recommendations are provided in Section 3 for additional liquefaction analyses to be conducted as design progresses, based on cases incorporating pore pressures during static conditions and ground shaking, as well as deeper failure planes and higher phreatic surfaces.

2.5 ADSP GUIDELINES

The response to RFI 008h Items #7d and #9a indicates that embankment design, construction, and management would be completed in accordance with the ADSP guidelines (ADNR 2017). It must be recognized that: 1) these are guidelines only; 2) they are not requirements, and are not intended to be a substitute for sound engineering analyses in accordance with the current state of practice for stability and seepage modeling, based on principles of soil mechanics, hydrology, and geotechnical and hydraulic engineering; and 3) KP, not ADSP, is the dam designer. The expected standard of care is outlined in the ADSP guidelines as follows: "Compliance with government regulations represents only a minimum standard of care;" and "Courts may assess a higher standard of care utilizing the 'reasonable person' standard and foreseeability of risk as the criteria." Therefore, the advanced designs must be completed in accordance with sound geotechnical analyses above and beyond any regulatory guidelines, and the burden of responsibility must be with the designer and not deferred to the ADSP guidelines.

The ADSP guidelines state that for FoS evaluations, the minimum allowable FoS is an important design criterion, and can vary by component and failure mode. For geotechnical slope stability, a well-defended minimum FoS of 1.5 may be adequate for a given condition, but for an underdrain, an FoS of 1.5 may be inadequate. Also, the FoS itself provides no correlation to failure probability or risk reduction. For example, the annual probability of a slope failure is 10^{-6} at an FoS of 1.5 for a project having a high level of engineering analysis, but for the same level of risk reduction (or annual failure probability) against internal erosion, the FoS must exceed 6 (Altarejos-Garcia et al. 2015), assuming the consequences from either failure mechanism are the same (ADNR 2017).

Specifying a minimum FoS has limited value unless parameters used to calculate it are defined. For example, Figure 13-1 in ADNR (2017) shows how FoSs for slope stability and level of engineering correlate with a subjective assignment of failure probability. The figure is based on FoS values defined as "shear strength along the sliding surface divided by shear stress along the same surface, [determined] in a manner consistent with its development." However, Silva et al. (2008) state that the figure "should not be used with a factor of safety defined as maximum allowable force divided by the applied force as this definition is not consistent with our method." In other words, when a reference is used for a minimum FoS calculation, the calculation method must be consistent with the reference, and the values used in the FoS calculation must be defined. A calculated FoS may be misleading relative to its risk reduction, depending on the level of engineering detail.

Silva et al. (2008) show that the level of engineering or detail has a greater influence on the failure probability than increasing the FoS. The level of detail refers to the amount of engineering from site investigations through design, construction, operations, and monitoring. In ADNR (2017) Figure 13-1, Category I projects represent the "best" level of engineering expected for projects with high failure consequences, whereas Category IV projects represent a "poor" level of engineering. The increased level of engineering effort and detail result in improved design that has greater capacity to resist the applied (demand) forces. The improved design serves to reduce the inherent uncertainty in the design performance, which reduces the failure probability for a given value of FoS. For a typical minimum FoS of 1.5, the best level of engineering reduces the failure probability by five orders of magnitude compared to a poorly engineered project. For a fixed set of consequences under either category, there is a direct correlation between risk reduction and level of detail.

The ADSP guidelines note that Figure 13-1 is based on geotechnical slope stability and specific FoS definition. Although the numerical values from Figure 13-1 and an associated table should not be extrapolated to other engineered components, the correlation between increasing the level of engineering detail and reducing the relative probability of failure translates to a reduction in risk posed by any engineered feature, even if that risk reduction cannot be quantitatively estimated.

2.6 Seismic Stability Methods

The response to Item #8 in RFI 008h refers to preliminary seismic deformation analyses of the bulk TSF main embankment using the Bray method (Bray and Travasarou 2007) (results presented in PLP 2019-RFI 008g). The Bray method is a simplified semi-empirical predictive relationship for estimating horizontal displacements due to earthquake-induced deformations. The method does not apply to cases where liquefaction or high pore pressures occur. Therefore, calculated displacements based on the Bray method should be considered an underestimation in the event of tailings liquefaction or high embankment pore pressures.

The stability of the bulk TSF main embankment based on deformations and global stability should be modeled and analyzed with full consideration of time-dependent pore pressure development and strength loss in both the tailings and embankment material during and after design earthquake loading. As described in the Item #8 response, PLP has committed to conducting numerical modeling as part of the design phase to estimate potential displacements within the structure. Recommendations are provided in Section 3.0 to specifically include the pore pressure conditions described above in the numerical analyses.

2.7 Embankment Stability with High Phreatic Surface

The response to Item #9c in RFI 008h (Figure 5) presents the results of a post-liquefaction, static stability analysis of the bulk TSF main embankment in the downstream direction assuming a higher phreatic surface than that used in Figure 4. However, the assumed phreatic surface could be even higher if clogging occurs in the embankment, potentially seeping out of the buttress face. Also, no pore pressures are assumed (i.e., ru = 0), which is not reasonable for an earthquake with a long duration of strong ground shaking. Thus, the calculated FoS and associated ky or deformations are conditional on relatively optimistic assumptions, which overstate embankment stability and underestimate risk if the planned embankment flow-through functionality does not perform as assumed.

In addition, as with the tailings liquefaction results described in Section 2.4 (RFI 008h Figures 2 and 4), the results for the high phreatic surface in the bulk TSF main embankment are provided for a post-liquefaction case only, but not for a case during the period of strong ground shaking.

To reiterate from Section 2.4, potential critical pore pressures in the bulk TSF main embankment cannot be assumed away by claiming either a favorable phreatic surface in the embankment or such highly permeable embankment material that excess pore pressures could not develop during design earthquake loading. A relatively high phreatic surface in the downslope embankment in combination with high excess pore pressures during ground motions are credible adverse conditions that must be considered as part of a technically defensible analysis of long-term TSF stability.

At this conceptual phase of the bulk TSF main embankment design, it is prudent for sensitivity analyses to be conducted to challenge the design and identify possible weaknesses for further design attention. Recommendations are provided in Section 3.0 for additional liquefaction analyses to be conducted as design progresses, by assuming higher phreatic surfaces in the embankment if flow-through is impeded in the rockfill, and analyzing higher pore pressures during ground shaking.

2.8 Comparison of Centerline Dam Examples to Pebble Bulk TSF Main Embankment

The response to Item #11 of RFI 008h (Table 1 – Summary of Comparable Centerline Dams) provides a list of 10 mine sites with centerline tailings dams that are considered comparable to the planned bulk TSF main embankment. Eleven tailings dams in total are listed because two tailings dams are listed under Highland Valley Copper. However, as described below, only three of these dams are directly comparable to the planned bulk TSF main embankment, three more are somewhat comparable, and the remaining five are not comparable.

Three dams in Table 1 are directly comparable to the planned bulk TSF main embankment with regard to centerline construction. The Constancia dam consists of zoned rockfill with a vertical clay core and is greater than 328 feet high. The Highland Valley H-H dam is described as an "earthfill dam with a low permeability vertical core, with random fill and tailings placed upstream and variable waste fill on the downstream side," and is 318 feet high. The Montana Resources dam is constructed of rockfill and is 750 feet high. These three dams have similar configurations and materials as planned for the bulk TSF main embankment. The Constancia and Highland Valley H-H dams are lower, and the Montana Resources dam is higher, than the planned bulk TSF main embankment. These dams are still being raised. All other dams in Table 1 are different than the planned main embankment as described below.

Three dams in Table 1 are described as "modified centerline" dams, or hybrids of centerline and upstream or downstream construction with rockfill raises. These are somewhat comparable to the planned bulk TSF main embankment configuration. The Alumbrera dam is described as rockfill/earthfill and is projected to be 540 feet high. The Alumbrera reference in Table 1 (Kostaschuk et al. 2000) describes a starter dam free-draining design, but has no information on the dam raises. Goldcorp, Inc. (2009) indicates that at the Alumbrera dam:

> Tailings from the process plant flow by gravity pipeline… to an engineered, centreline dam…. Distribution is affected by spigotting along the upstream face of the dam. Supernatant water is pumped back to the process plant and seepage is collected downstream of the dam and pumped back. The dam is raised using waste rock with a core of selected material and remains a significant capital cost throughout the life of the mine.

The Fort Knox dam is rockfill and 350 feet high, and had planned centerline raises but was raised as a downstream-to-centerline hybrid. The Montana Tunnels dam consists of rockfill, was permitted to 410 feet in 2008, and started downstream with raises closer to upstream than centerline.

Five of the dams in Table 1 are described as being raised by using cyclone sand instead of rockfill, which means that the dams are not comparable to the planned bulk TSF main embankment. These are the Brenda, Cerro Verde, Gibraltar, Highland Valley L-L, and Thompson Creek dams, all of which have rockfill or earthfill starter dams followed by cyclone sand raises, and actual or permitted heights ranging from 385 to 985 feet. In addition, the Gibraltar TSF was raised by the upstream method, not centerline (Klohn Crippen Berger Ltd. [KCB] 2014). Several of these cyclone sand dams have reported sand boils or sinkholes, such as the Brenda TSF seepage and sand boils (Klohn 1984); Gibraltar dam sinkholes and internal erosion (KCB 2018a); and seepage and exit erosion from the sand fill toe at the Highmont part of Highland Valley dam (KCB 2018b). Regardless, these are not relevant to the planned bulk TSF main embankment that would not be raised using cyclone sands.

A centerline tailings dam that is very similar to the planned bulk TSF main embankment but is not in Table 1 is the Mount Polley dam in British Columbia, part of which failed in August 2014 because of a deep clay layer that was mischaracterized, and an over-steep downstream slope. The Mount Polley review panel (Morgenstern et al. 2015) also identified other factors that could have resulted in failure in the future: an eroding filter/transition zone, and water near the dam for long periods of time. This points to the importance of ensuring a stable filter/transition zone, proper tailings and water management, and conducting stability analyses that test upset conditions related to these elements to identify vulnerabilities in the design.

2.9 DOWNSTREAM ALTERNATIVE

The response to Item #12 in RFI 008h provides a qualitative comparison between the stability of 1) a downstream-constructed embankment assuming deep tailings liquefaction, and 2) the results for the planned modified centerline embankment for the bulk TSF main embankment discussed in the above sections. The conclusion that they would be similar is not supported with a modeled slope stability analysis in this RFI; however, it is acknowledged that a comparable post-liquefaction analysis for the downstream alternative assuming full depth of tailings liquefaction was provided in a subsequent RFI response, PLP 2019-RFI 130. The results of this analysis are similar to those of the modified centerline alternative in RFI 008h, Item #7c (Figure 4).

3.0 CONCLUSIONS AND RECOMMENDATIONS

This technical memorandum provides a review of items in the response to RFI 008h regarding liquefaction potential of the bulk TSF tailings and related stability of the centerline portion of the bulk TSF main embankment. Several areas of uncertainty are identified that should be disclosed during the EIS process, and resolved during additional investigation and analysis as the bulk TSF and embankment designs progress during state permitting and final design:

- It is uncertain that thickened tailings at 55 percent solids would segregate enough to promote reduction of the phreatic surface near the bulk TSF main embankment, which translates to uncertainties regarding the effect of tailings segregation on embankment stability. Future testing and analysis committed to by PLP in RFI 008h would further the understanding of tailings deposition behavior.
- There is concern that some and perhaps all of the entire centerline part of the bulk TSF main embankment (not just the uppermost raise) could slide into potentially undrained tailings and have consequent effects in a downstream direction. Future stability analyses planned during detailed design would reduce these uncertainties, but should consider incorporating the recommendations listed below.
- The central concern is the overall main embankment and TSF stability. The critical combination of excess pore pressures and ground shaking, as well as static liquefaction without any ground shaking to induce it, either of which could result in minimum embankment stability, may occur at any time. Therefore, embankment stability must be appropriately analyzed reflecting both pore pressure buildup (strength loss) in tailings and embankment materials, under both static conditions and with continued ground motion during the duration of ground shaking based on design earthquake time history input motions.
- Potential critical pore pressures in the bulk TSF main embankment should not be "assumed away" by claiming either only a favorable (very low or deep) phreatic surface in the embankment, or such highly permeable embankment material that excess pore pressures (above hydrostatic) could not develop during earthquake loading. A relatively high phreatic surface in the downslope embankment in combination with high excess pore pressures are credible adverse conditions that must be considered as part of a long-term TSF stability analysis.

We recommend that, in addition to future testing and analyses that PLP has committed to in RFI 008h, the following be considered as mitigation in the EIS:

- Emergency procedures for dealing with water levels in the bulk TSF that approach or exceed defined maximum operating levels should be incorporated into the EAP and should be completed and available prior to the start of construction.
- As the bulk TSF main embankment design progresses based on additional investigation and testing, stability analyses of the upstream and downstream slopes that incorporate tailings liquefaction and higher embankment pore pressures should be rerun. These should:
    - Continue to assume tailings liquefaction depths up to at least the full depth of the centerline portion of the embankment.
    - Continue to include tailings liquefaction under static conditions.
    - Include a tailings liquefaction case during an earthquake (not just after), when strong ground motions cause pore pressures to increase leading to liquefaction;

the seismic cases should be based on a full time-dependent dynamic analysis that includes pore pressure effects (e.g., FLAC[1] or equivalent) implemented for each applicable earthquake time history (described in RFI 008h, Item #4).

- Include cases assuming deeper slide planes in the centerline raises (not just the uppermost raise) and through the full embankment section, that evaluate potential effects in both upstream and downstream directions.
- Assume shallower phreatic surfaces in the embankment, including cases where flow-through is impeded in the rockfill and seeps out of the face of the embankment.

[1] Fast Lagrangian Analysis of Continua (FLAC) numerical modeling software

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ATTACHMENT A – RESPONSE TO RFI 008H

RFI 008h
Pebble Project EIS

Request for Information

Title/Subject:	**Followup on Seismic Stability Analysis**
Requestor:	**AECOM**
Date Transmitted:	**7/19/19**
Recipient:	**Pebble Limited Partnership (PLP)**
Response Requested by:	**8/9/19**
Rationale:	**The two PLP responses to RFI 008g (received 6/10/19 and 7/9/19) provided supporting analyses to address public comments received from J. Kuenzli (3/21/19) on the bulk TSF main embankment seismic stability analyses in the DEIS. These comments were primarily concerned with seismic hazard analysis of the main embankment and seismic stability of the upstream face of the centerline-constructed part of the main embankment. A review of the two responses to RFI 008g, plus new comments received from B. Santana (5/30/19) has triggered the need for this RFI 008h for additional information to assess the earthquake parameters used in the seismic hazard analyses, and the range of potential effects on embankment stability in the event of deep liquefaction of the tailings.**
Describe the Information Requested and Level of Detail:	**<u>RFI 008g, Item #1: Seismic Hazard Analyses.</u> The following questions pertain to review of information under Item #1 in the initial RFI 008g response and the Knight Piésold (KP 2019a) *Report on Seismicity Assessment and Seismic Design Parameters* provided with the second RFI 008g response, which is an update of the KP's (2013) seismic report.** **1) KP (2019a) uses new ground-motion models (GMMs) for the maximum credible earthquake (MCE) in the updated deterministic seismic hazard analysis (DSHA). For shallow crustal earthquakes, the newer GMMs used were the five Next Generation Attenuation (NGA) West2 equations (Bozorgnia et al. 2014), which were described in RFI 008c. KP (2019a) used all five equations and gave them the same weights[1] that the US Geological Survey (USGS) assigned in its 2014 national seismic hazard study (Peterson et al. 2014). However, for the current code cycle the USGS has dropped the Idriss equation because it doesn't include Vs30[2] and basin depth terms. What is the effect on the 2019 ground motions by dropping the Idriss equation from the DSHA?** **2) Subduction Zone Earthquakes:** **a. KP (2019a) uses an equation from Abrahamson et al. (2016) for MCE ground motions from subduction zone earthquakes. The Pacific Earthquake Engineering Research Center (PEER) recently published its first release of the NGA subduction GMMs for interface and intraslab subduction earthquakes (Abrahamson et al. 2018). The PEER Excel workbook with this GMM[3] can be used in the DSHA in place of the Abrahamson et al. (2016) equation to check its effect on the MCE ground motions from the postulated subduction zone earthquakes. How does this affect the DSHA results?** **b. The response spectra KP (2019a) computed for the subduction scenario earthquakes only extend to 2 seconds. The Atkinson and Boore (2003) and Zhao et al. (2006) equations extend to 3- and 5-second periods, respectively. Please provide the response spectra computed to 10 seconds by noting the applicable period bands for each equation and re-weighting the remaining equations when one is no longer applicable.** **c. Three other NGA subduction equations are scheduled to be released later this summer or fall 2019. Does PLP plan to complete another update of the DSHA following this?** **3) The USGS plans to update the Wesson (2007) seismic source model and the ground motions associated with this model in 2023. However, this**

	model could be programmed into current Probabilistic Seismic Hazard Analysis (PSHA) software, such as EZ-FRISK, which includes the recent NGA West2 GMMs and several of the more recent subduction GMMs. Thus, the PSHA using this software could generate updated OBE (475-yr) response spectra, and response spectra for other return periods at the mine and port sites, in the interim period between now and the release of the updated USGS Alaskan model in 2023. Please consider doing this interim update of the PSHA results in an updated version of the seismic report. 4) Acceleration time histories will be required as input excitation for nonlinear response history analyses of all mine site embankments for the MCE. Please provide the acceleration time histories that are proposed for the deterministic earthquake scenarios judged to potentially produce the maximum responses of all of the embankments. State how many time histories will be provided for each scenario, and how the selected time histories will be modified to be compatible with the target 84th percentile response spectrum computed for each scenario.[4] RFI 008g, Item #2: Tailings Liquefaction and Seismic Stability of Upstream Face. The following requests pertain to information provided under Item #2 of the initial RFI 008g response and the KP (2019b) memo titled *Main Embankment Stability Assessment – Static and Post-liquefaction*, attached to the second RFI 008g response. 5) Item #2a, 2nd paragraph, of the initial RFI 008g response indicates that the tailings beaches adjacent to the upstream face would be drained under normal operating pond conditions, that pore pressures would be monitored, and that placement of fill on tailings may be modified in the event of pore pressure development during construction to allow the pore pressure to dissipate. Please provide further descriptions of: a. The confidence levels that the tailings will segregate with coarser fractions nearer the main embankment, and that the beaches can be continually drained to achieve uninterrupted flow-through seepage out of the TSF as a part of normal operating conditions; b. What specific tailings and embankment operational practices would be employed to ensure that the tailings will segregate and that the flow-through drainage occurs; c. What specific additional tailings analyses would be conducted prior to final design to confirm the assumed tailings segregation and drainage behaviors; and more detail as to what mitigation steps would be taken and what material and equipment would be available on site in accordance with execution of the observational method if excess pore pressures develop and are sustained; and d. How tailings placement procedures and TSF operations might be modified if a potentially critical situation arises with respect to an inability to remove water from the TSF in a sufficiently expeditious manner to avoid overtopping the embankment or compromising the stability of the embankment. 6) Item #2f, 2nd paragraph refers to undrained strength parameters that were obtained from published values and assumed in the analysis. Typically these would be determined and confirmed from site-specific tailings field and laboratory analyses. Indicate which of these parameters are based on site-specific data, if available, and provide that data. Describe plans for conducting these analyses and at what phases of design and permitting this would be completed, the liquefaction analyses updated, and if necessary, redesign or refinement of raise construction addressed. 7) Depth of Liquefaction: Item #2g and KP (2019b) provide the results of upstream stability assessments based on an assumption that the tailings would liquefy to a depth no deeper than 100 ft. a. While this 100-ft depth criteria is consistent with general past industry practice, many publications in the literature state that new

	and loose soils, such as tailings that are geologically young deposits, should be considered as being potentially liquefiable at greater depths, and some of these publications suggest tailings could liquefy to much greater depths than 100 ft (described in DEIS Appendix K3.15). Please provide references that clearly describe the depth of liquefaction constraints, noting that the Kramer (1996) and Geo-Slope (2018) references cited in the initial RFI 008g response do not appear address the potential depth of liquefaction, and that the Kramer reference is now almost 25 years old and has been superseded by several newer and recent publications. b. Please provide additional stability analyses similar to the cases provided in the initial RFI 008g response and KP (2019b) that evaluate stability of the full embankment section assuming liquefied (affected) tailings to the full depth of the vertical upstream face. This should include the following two cases: (1) during the period of strong ground shaking causing increased pore pressures leading to liquefaction; and (2) immediately after strong ground shaking has ceased with full liquefaction (affected tailings at post-liquefaction residual strength). c. At this early conceptual point of the design process, it is prudent to check on the potential resilience of the current design concept with respect to tailings liquefaction. As a kind of index of that resilience, please conduct a stability analysis of the embankment by assuming that the tailings have liquefied to their total depth and are at their assumed residual shear strength (USR=0.05), then calculate the static factor of safety (FoS) and yield acceleration. For the current design concept to have credibility for advancing it to more detailed design, a calculated static FoS must exceed 1.0 and the yield acceleration (ky) must exceed ½ the peak ground acceleration (PGA) of the design earthquakes. d. On the basis of the above items #7a, b, and c, the potential liquefaction of the deeper tailings under and adjacent to the footprint of the upstream parts of the centerline raises is a concern with respect to the stability of the upstream slopes, and potentially of the raises themselves. A concern is that deep tailings could suddenly liquefy under static or dynamic (earthquake) loading, causing a containment failure and release that cannot be practically mitigated in a timely way. The RFI 008 series documents reduce but do not eliminate that risk. Therefore, given what is known and not known, it may be not be reasonable to preclude nor even quantify that potential. The practical bottom line is that the risk is undefined, but high. Please describe how the embankment planning, design, construction, and operations would be conducted to reduce this risk. 8) Also at this point of the design process, it is prudent to conduct sensitivity analyses on the yield acceleration (ky) criterion. For example, ky could be reduced to ¼ PGA or arguably even a bit lower (increasing potential deformation). The ratio of ky/PGA can be considered an index of potential embankment displacement, with the potential displacement increasing with decreasing ky/PGA and vice versa. There is no fundamentally correct criterion. A ky/PGA=0.50 could be considered as "too conservative," allowing "too little displacement," so not acceptable. A lower value, like ky/PGA=0.25, would be less open to being considered "too conservative to be acceptable." Please conduct sensitivity analyses to compare calculated displacements as a function of ky/PGA before suggesting an acceptable value. 9) Case 3 in KP (2019b) assumes that the phreatic surface would be held at or near the bottom of the main embankment, and that there would be no earthquake-induced pore pressures in embankment materials. However, it is possible that the embankment bulk, filter, and transition zone materials could impede the flow-through concept because of

	incompatible gradations, particle deterioration, filter failure, chemical precipitation, etc., thereby causing the phreatic surface in the embankment to rise and the FoS to decrease. Please provide: a. Backup documentation that supports the assumption that the phreatic surface would be near the bottom of the embankment and stay there for the duration of the TSF operations and into closure and post-closure; b. A discussion of the confidence level in the rockfill gradations and quality expected to be produced from on-site quarries and the ability to maintain these gradations and resist particle degradation; and c. Conduct sensitivity analyses that assume a much higher phreatic surface in the embankment. 10) The initial RFI 008g response refers to allowable and calculated FoS in several places. As is known in the geotechnical engineering practice and described in Alaska Dam Safety Program guidelines (ADNR 2017), a calculated FoS is only as reliable as the quality of the data that the calculation is based on and the level of engineering analyses completed. Please discuss the confidence levels of each of the allowable and calculated FoS in RFI 008g with respect to the confidence of their underlying assumptions, level of engineering analyses completed, and published data versus site-specific data. 11) Provide examples of similar size and shape tailings embankments built by centerline construction methods worldwide where they have worked successfully before and are still working successfully. Describe operational issues at similar facilities related to tailings liquefaction potential and embankment stability, and how they were addressed. 12) A main part the NEPA analysis is to compare alternatives. Please discuss the impacts that deep tailings liquefaction in the bulk TSF would have on the main embankment and on the TSF in general if the main embankment was built by downstream construction methods to its full ultimate height. Notes: 1. i.e., 0.12 to the Idriss equation and 0.22 each to the other four equations 2. Time-averaged shear-wave velocity to 30 m depth 3. The equations in the Abrahamson et al. (2018) report have a couple of typographical errors and should not be used. 4. Representative accelerograms can be obtained for the shallow crustal earthquake scenarios through the PEER web-search tool. A similar tool for subduction zone accelerograms under development, but accelerograms recorded during the 2011 M9 Tohoku, Japan and the 2010 M8.8 Maule, Chile megathrust events are available, for example, through the Center for Earthquake Strong Motion Data (https://www.strongmotioncenter.org/). Synthetic accelerograms for a number of simulated M9 earthquakes on the Cascadia megathrust are also available. References: Abrahamson, N.A., N. Gregor, K. and Addo. 2016. BC Hydro Ground Motion Prediction Equations for Subduction Earthquakes. Earthquake Spectra, Vol. 32, p. 23-44. Abrahamson, N., N. Kuehn, Z. Gulerce, N. Gregor, Y. Bozorgnia, G. Parker, J. Stewart, B. Chiou, I.M. Idriss, K. Campbell, and R. Youngs. 2018. Update of the BC Hydro Subduction Ground-Motion Model using the NGA-Subduction Dataset. Pacific Earthquake Engineering Research Center (PEER), University of California, Berkeley. PEER Report No. 2018/02, June 2018. ADNR. 2017. Guidelines for Cooperation with the Alaska Dam Safety Program. Draft Revision, July 2017. Atkinson, G.M. and D.M. Boore. 2003. Empirical Ground-Motion Relations for Subduction-Zone Earthquakes and Their Application to Cascadia and Other

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Recipient Response Form

Date Received from USACE:	Click here to enter text.

Response from Recipient (Describe Information Requested to the	**<u>RFI 008g, Item #1: Seismic Hazard Analyses.</u> The following questions pertain to review of information under Item #1 in the initial RFI 008g response and the Knight Piésold (KP 2019a) *Report on Seismicity Assessment and Seismic Design Parameters* provided with the second RFI 008g response, which is an**

Level of Detail Requested; Provide Attachments as Needed):	**update of the KP's (2013) seismic report.** **1) KP (2019a) uses new ground-motion models (GMMs) for the maximum credible earthquake (MCE) in the updated deterministic seismic hazard analysis (DSHA). For shallow crustal earthquakes, the newer GMMs used were the five Next Generation Attenuation (NGA) West2 equations (Bozorgnia et al. 2014), which were described in RFI 008c. KP (2019a) used all five equations and gave them the same weights[1] that the US Geological Survey (USGS) assigned in its 2014 national seismic hazard study (Peterson et al. 2014). However, for the current code cycle the USGS has dropped the Idriss equation because it doesn't include Vs30[2] and basin depth terms. What is the effect on the 2019 ground motions by dropping the Idriss equation from the DSHA?** It is understood that the USGS dropped the Idriss equation for its latest code cycle because it did not include the ability to predict peak ground motions for softer soil site conditions (Vs30 < 450 m/sec) and did not include basin depth terms, which are both requirements for the 2018 National Seismic Hazard Maps. These omissions do not impact the Pebble seismic hazard assessment. Petersen et al. (2018) state that exclusion of the Idriss model does not reflect on the quality of the model. Consequently, it is considered appropriate to maintain the Idriss equation. If any loose/soft soil conditions are encountered during the design process it will be examined in more detail by conducting site response analysis and/or by remediation (e.g. excavation of soft soils). For any cases where it is required to calculate preliminary ground motion parameters for sites with a Vs30 < 450 m/sec the NGA West2 equations will be used (with equal weighting) but with the Idriss equation omitted. Dropping the Idriss equation from the DSHA results in the following changes to the predicted 84th percentile PGA values for the two shallow crustal MCE scenarios defined for the mine site in KP (2019a): • M6.5 maximum background earthquake, minor PGA increase from 0.56g to 0.58g. • M7.5 on mapped Lake Clark fault, negligible PGA decrease from 0.32g to about 0.31g (0.315g). **2) Subduction Zone Earthquakes:** **a. KP (2019a) uses an equation from Abrahamson et al. (2016) for MCE ground motions from subduction zone earthquakes. The Pacific Earthquake Engineering Research Center (PEER) recently published its first release of the NGA subduction GMMs for interface and intraslab subduction earthquakes (Abrahamson et al. 2018). The PEER Excel workbook with this GMM[3] can be used in the DSHA in place of the Abrahamson et al. (2016) equation to check its effect on the MCE ground motions from the postulated subduction zone earthquakes. How does this affect the DSHA results?** The available Excel workbook provided by PEER for the Abrahamson et al (2018) ground motion model has been used to calculate peak ground motions for the M8 intraslab subduction and M9.2 interface subduction MCE events defined in the KP 2019 seismic hazard assessment. The predicted values of peak ground acceleration and spectral accelerations (defining the response spectrum) are significantly lower for the M8 intraslab subduction MCE compared to the values calculated using Abrahamson et al (2016). For the M9.2 interface subduction event, the calculated peak ground motions are significantly lower for the peak ground acceleration and short period spectral accelerations (periods less than 0.5 seconds). For longer period spectral accelerations, the values are very similar to those given by Abrahamson et al

	(2016), particular over the period range of 0.5 to 1.0 seconds. Embankment stability analyses completed to date have demonstrated that the period range of interest for predicting seismically induced deformations is within 0.1 to 1.0 seconds for the mine site dam structures.

It is noted that the ground motion model of Abrahamson et al (2018) was developed for application to the Cascadia region for updating the 2020 US national hazard maps. The NGA Subduction ground motion models will supersede the Abrahamson et al (2018) model.

The higher (more conservative) ground motions presented in Knight Piésold (KP 2019a) Report on Seismicity Assessment and Seismic Design Parameters will be maintained until the NGA Subduction ground motion models are available.

b. The response spectra KP (2019a) computed for the subduction scenario earthquakes only extend to 2 seconds. The Atkinson and Boore (2003) and Zhao et al. (2006) equations extend to 3- and 5-second periods, respectively. Please provide the response spectra computed to 10 seconds by noting the applicable period bands for each equation and re-weighting the remaining equations when one is no longer applicable.

The deterministic response spectra provided on Figure 4.1 and in Table 4.2 of KP 2019a have been revised to include spectral acceleration values up to 10 seconds as shown in Figure 1 and Table 1 below. The spectral accelerations for periods from 3.0 to 5.0 seconds have been calculated using average values (equal weighting) using the ground motion prediction equations of Zhao et al (2006) and Abrahamson and Addo (2016). Spectral accelerations for periods of 7.5 and 10.0 seconds have been calculated using only Abrahamson and Addo (2016).

Table 1 Deterministic Response Spectra for TSF Design Earthquake Scenarios

Spectral Period (seconds)	Spectral Acceleration (g)			
	Magnitude 9.2 Interface Subduction	Magnitude 8.0 Intraslab Subduction	Magnitude 7.5 Shallow Crustal Fault	Magnitude 6.5 Shallow Crustal Fault
PGA	0.16	0.61	0.32	0.56
0.02	0.161	0.606	0.320	0.576
0.03	-	-	0.354	0.638
0.05	0.192	0.877	0.449	0.823
0.075	0.235	1.093	0.577	1.078
0.10	0.283	1.310	0.652	1.247
0.15	0.342	1.391	0.720	1.405
0.20	0.334	1.251	0.698	1.349
0.25	0.335	1.125	0.645	1.224
0.30	0.344	0.985	0.594	1.093
0.40	0.362	0.744	0.507	0.887
0.50	0.260	0.600	0.438	0.733
0.75	0.226	0.419	0.311	0.485
1.0	0.210	0.332	0.234	0.349
1.5	0.148	0.237	0.154	0.198
2.0	0.104	0.187	0.113	0.130

3.0	0.047	0.082	0.075	0.069
4.0	0.033	0.053	0.056	0.042
5.0	0.023	0.034	0.045	0.030
7.5	0.014	0.013	0.028	0.014
10.0	0.010	0.008	0.018	0.008

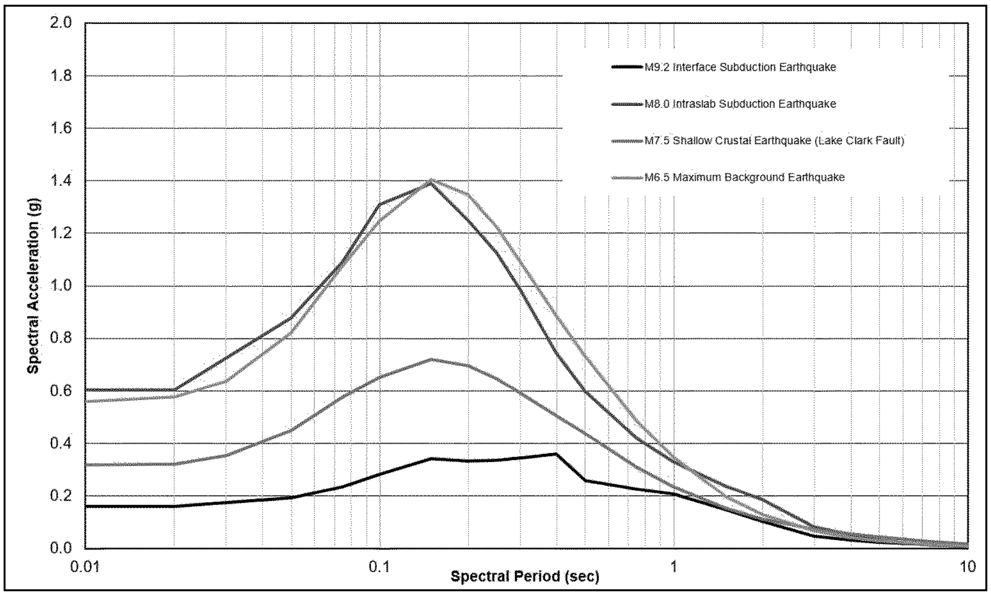

Figure 1 – Deterministic Response Spectra for TSF Maximum Design Earthquake Scenarios

c. **Three other NGA subduction equations are scheduled to be released later this summer or fall 2019. Does PLP plan to complete another update of the DSHA following this?**

The DSHA will be updated for future report revisions and design analyses using the NGA subduction ground motion prediction equations, once they have been formally released and reviewed.

3) **The USGS plans to update the Wesson (2007) seismic source model and the ground motions associated with this model in 2023. However, this model could be programmed into current Probabilistic Seismic Hazard Analysis (PSHA) software, such as EZ-FRISK, which includes the recent NGA West2 GMMs and several of the more recent subduction GMMs. Thus, the PSHA using this software could generate updated OBE (475-yr) response spectra, and response spectra for other return periods at the mine and port sites, in the interim period between now and the release of the updated USGS Alaskan model in 2023. Please consider doing this interim update of the PSHA results in an updated version of the seismic report.**

The tailings dams (and other dam structures) will be designed for the selected deterministic MCE scenarios defined for the mine site. Current design work shows that these events control the design of the dams, not the OBE. Any revision to the probabilistically-derived seismic parameters defining the OBE will not have an impact on the design of the dams, or increase predicted seismic deformations to a level that results in the dams not satisfying performance objectives.

Deaggregation information for the current PSHA provided by the USGS indicates that the subduction earthquakes contribute approximately 90% of the seismic

	hazard for return periods of 475 and 2475 years. Consequently, using the latest NGA West2 ground motion models for shallow crustal earthquakes will have a minor impact on the predicted peak ground motions. Also, as discussed above (Question 2a.) a review of the recently published subduction ground motion models of Abrahamson et al (2018) indicates that predicted peak ground motions are likely to be lower using this ground motion model compared to those used in the current USGS seismic hazard model for Alaska. Consequently, it is expected that peak ground motions predicted by an updated PSHA using the latest available ground motion models will be similar or lower. The seismic source model for Alaska provided with EZ-FRISK has been reviewed to consider its use to provide an updated PSHA using more recent ground motion models. The use of EZ-FRISK (or other PSHA software) will be used for an interim update of the PSHA if it is deemed appropriate and of use to design development. The seismic parameters (including response spectra) currently defined for structural design (mine and port sites) are consistent with current code requirements and design parameters (which are based on the Wesson (2007) model for Alaska). Note that these parameters are not included in KP 2019a. **4) Acceleration time histories will be required as input excitation for nonlinear response history analyses of all mine site embankments for the MCE. Please provide the acceleration time histories that are proposed for the deterministic earthquake scenarios judged to potentially produce the maximum responses of all of the embankments. State how many time histories will be provided for each scenario, and how the selected time histories will be modified to be compatible with the target 84th percentile response spectrum computed for each scenario.[4]** Acceleration time-history records will be sourced and selected to represent each of the four MCE scenarios defined for the mine site. This will include consideration of the earthquake magnitude, rupture mechanism, focal depth and source directivity (for the near-field shallow crustal events), in addition to the recording site geology, topography and recording instrument location. A minimum of 8 earthquake time-history records will be adopted for each design MCE. It is likely that sufficient earthquake records will be available (from PEER and other time-history web-based databases) to provide a minimum of 8 records representing the M6.5 and M7.5 shallow crustal earthquake MCE scenarios. Fewer earthquake records representing the M9+ interface subduction and M8 intraslab subduction earthquake may be available (fewer time-history records available for these types of earthquake). However, it is anticipated that acceleration time-history records provided by the M9 Tohoku, Japan earthquake in 2011 and the M8.8 Maule, Chile earthquake in 2010 will be suitable for representing the M9.2 interface subduction event. Web-based search tools for finding and selecting subduction earthquakes will also be used when available. If required, synthetic acceleration time-histories (including those developed for the Cascadia subduction zone) will be used to supplement natural earthquake records to provide the minimum number of 8 records. Each of the selected earthquake records will be modified by scaling and spectrally matching to the corresponding MCE response spectrum. **<u>RFI 008g, Item #2: Tailings Liquefaction and Seismic Stability of Upstream Face.</u> The following requests pertain to information provided under Item #2 of the initial RFI 008g response and the KP (2019b) memo titled *Main Embankment Stability Assessment – Static and Post-liquefaction*, attached to the second RFI 008g response.**

	5) Item #2a, 2nd paragraph, of the initial RFI 008g response indicates that the tailings beaches adjacent to the upstream face would be drained under normal operating pond conditions, that pore pressures would be monitored, and that placement of fill on tailings may be modified in the event of pore pressure development during construction to allow the pore pressure to dissipate. Please provide further descriptions of: **a. The confidence levels that the tailings will segregate with coarser fractions nearer the main embankment, and that the beaches can be continually drained to achieve uninterrupted flow-through seepage out of the TSF as a part of normal operating conditions;** It is well known in conventional tailings deposition that sedimentation processes occur as tailings slurry is discharged into a TSF. A discussion on this sedimentation and particle size sorting process presented in Vick (1990) is summarized as follows: Studies on particle size sorting that occur along the tailings beach indicate that as particles settle from the slurry, particles are transported along the beach surface by saltation and rolling. Hydraulic separation results in a tendency of finer particles on the beach to be carried and deposited further from the point of discharge. The degree of sorting depends on the gradation characteristics of the whole tailings discharge, where slurries with a wide range of particle sizes are more likely than slurries with poorly graded materials to exhibit beach grain-size segregation. The slurry tailings discharge as envisioned for the Bulk TSF results in deposition of the coarse fraction of tailings nearest to the discharge point and the finer tailings extending further into the facility. Maintaining the coarse fraction of the tailings beach against the embankment and implementation of appropriate filter relationships between the embankment materials and tailings beach will allow for the TSF to operate as a drained facility. A tailings deposition plan, to be included in the operations, maintenance and surveillance (OMS) manual, will be completed prior to operations. The OMS manual will include operating requirements, such as minimum beach widths to control the location of the supernatant pond and promote beach development. **b. What specific tailings and embankment operational practices would be employed to ensure that the tailings will segregate and that the flow-through drainage occurs;** The tailings deposition plan will be developed to limit the settlement of fine tailings near the embankment structures by controlling the location and duration of active discharge spigots. Alternating the tailings discharge location will allow for continuous development of the tailings beach and will control the extents of the coarse and fine tailings fractions. Regular discharge from the Main and South Embankments will be required to establish and maintain the coarse tailings beach adjacent to these embankments. The development of the tailings beach will continue throughout the mine life, with the tailings deposition being modified as required throughout operation based on beach development and the operating conditions on site. **c. What specific additional tailings analyses would be conducted prior to final design to confirm the assumed tailings segregation and drainage behaviors; and more detail as to what mitigation steps would be taken and what material and equipment would be available on site in accordance with execution of the observational method if excess pore pressures develop and are sustained; and** The tailings testing program, which is expected to be completed during the preliminary design phase of the Alaska Dam Safety Program, will include index testing to enable geotechnical classification of the materials, slurry settling, air drying, consolidation and permeability testing to determine the characteristics the

	tailings. This testing will occur under a range of conditions to be representative of expected field conditions. Results from this testwork will be used to validate the sensitivity analyses and material parameters used in the seepage analysis completed to date. Response to RFI 008g stated that if excess pore pressures develop in the tailings material adjacent to the upstream edge of the Zone U material as a result of construction in this area, the procedure of placing fill material on the tailings may be modified to allow the pore pressures to dissipate prior to continuing with fill placement. This may include a temporary stoppage of fill placement at that location. Piezometers installed in the tailings mass will monitor pore pressure readings during fill placement, and trigger levels will be established to monitor the development and dissipation of pore pressures to assist with constriction activities. These piezometers will be in place throughout the construction and operations of the facility. **d. How tailings placement procedures and TSF operations might be modified if a potentially critical situation arises with respect to an inability to remove water from the TSF in a sufficiently expeditious manner to avoid overtopping the embankment or compromising the stability of the embankment.** An emergency action plan will be defined as part of the OMS manual will include maximum pond operating levels for the TSFs, and a response plan to be implemented if the water levels exceed the defined maximum operating levels. The response plan may include adding additional pumping capacity to the reclaim system, or reducing/stopping the tailings discharge rate to the TSF until the water level has been returned to below the maximum operating level. The TSF concept includes additional operating storage volume above the maximum operating pond levels to provide complete containment of the Probable Maximum Flood, plus freeboard to reduce the likelihood of overtopping the TSF. **6) Item #2f, 2nd paragraph refers to undrained strength parameters that were obtained from published values and assumed in the analysis. Typically these would be determined and confirmed from site-specific tailings field and laboratory analyses. Indicate which of these parameters are based on site-specific data, if available, and provide that data. Describe plans for conducting these analyses and at what phases of design and permitting this would be completed, the liquefaction analyses updated, and if necessary, redesign or refinement of raise construction addressed.** The undrained strength parameters assigned for the preliminary analysis completed and reported in KP 2019b were selected using published values. Site-specific tailings teswork will be completed to support the preliminary design phase of the ADSP. Results from this testwork will be used to validate the material parameters, and if required, update the liquefaction analysis and embankment design during the preliminary and detailed design phases. The testwork will include index testing to enable geotechnical classification of the materials, slurry settling, air drying, consolidation and permeability testing, and strength testing to determine the characteristics the tailings. **7) Depth of Liquefaction: Item #2g and KP (2019b) provide the results of upstream stability assessments based on an assumption that the tailings would liquefy to a depth no deeper than 100 ft.** **a. While this 100-ft depth criteria is consistent with general past industry practice, many publications in the literature state that new**

and loose soils, such as tailings that are geologically young deposits, should be considered as being potentially liquefiable at greater depths, and some of these publications suggest tailings could liquefy to much greater depths than 100 ft (described in DEIS Appendix K3.15). Please provide references that clearly describe the depth of liquefaction constraints, noting that the Kramer (1996) and Geo-Slope (2018) references cited in the initial RFI 008g response do not appear address the potential depth of liquefaction, and that the Kramer reference is now almost 25 years old and has been superseded by several newer and recent publications.

The preliminary tailings liquefaction analysis was completed using the above-mentioned references (Kramer, 1996 and Geo-Slope, 2018) limiting the depth of liquefaction to 100 ft. The maximum depth of liquefaction stated in literature as 100 ft is based on the assumption that at depths greater than 100 ft, the void ratio of any material will be of sufficient magnitude to provide a soil buoyant unit weight that is higher than the potential porewater pressures due to earthquake loading could cause below 100 ft. Answer to 7b provides updated upstream stability results based on full tailings liquefaction.

b. Please provide additional stability analyses similar to the cases provided in the initial RFI 008g response and KP (2019b) that evaluate stability of the full embankment section assuming liquefied (affected) tailings to the full depth of the vertical upstream face. This should include the following two cases: (1) during the period of strong ground shaking causing increased pore pressures leading to liquefaction; and (2) immediately after strong ground shaking has ceased with full liquefaction (affected tailings at post-liquefaction residual strength).

The upstream post-liquefaction stability assessment considering full liquefaction of the tailings material (Case 2) was completed using the material Parameters defined in Table 2.

Table 2 Tailings Material Parameters

Material	Shear Strength	Unit Weight
		pcf
Post-Liq. Tailings	— = 0.05	90

A potential slip surface for the fully liquified case is shown on Figure 2. A slip surface with a FoS of 1.2 is shown, as per the minimum acceptance criteria. The results illustrated on Figure 2 indicate that the Zone U material placed on tailings beach deposits will likely undergo settlement, however, the deformations are likely to be constrained to the upstream zone of the embankment. These results are similar to the previous scenarios (KP, 2019b) where only 100 ft of tailings liquefaction was considered, as presented on Figure 3.

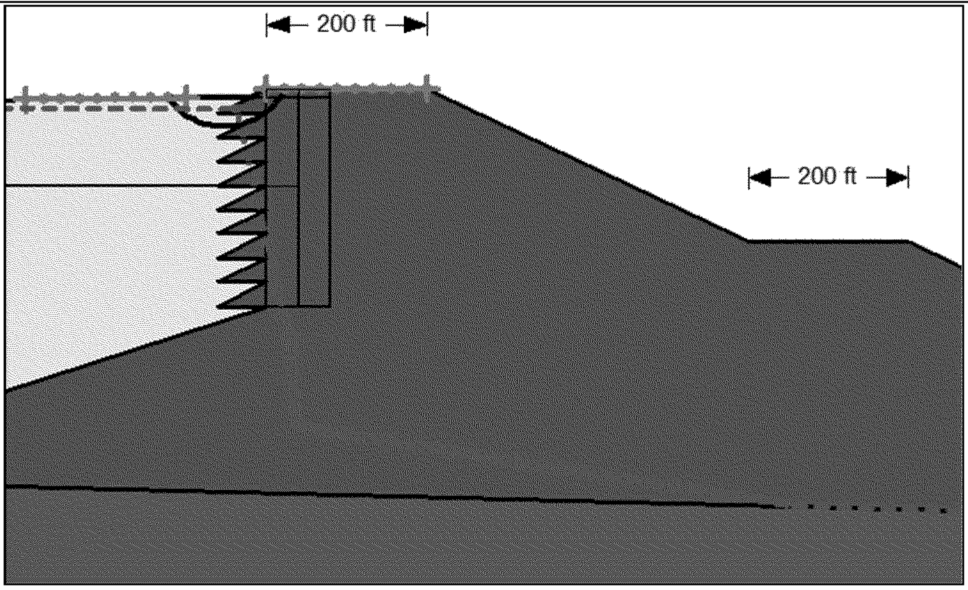

Figure 2 - Upstream Stability – Post Liquefaction – Full Depth

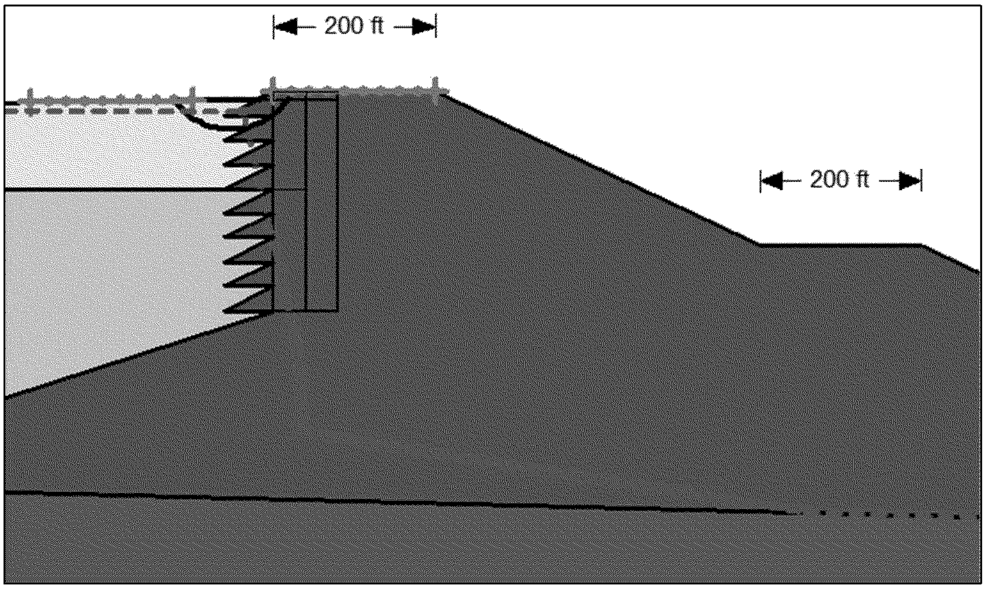

Figure 3 - Upstream Stability – Post Liquefaction – 100 ft Depth

c. **At this early conceptual point of the design process, it is prudent to check on the potential resilience of the current design concept with respect to tailings liquefaction. As a kind of index of that resilience, please conduct a stability analysis of the embankment by assuming that the tailings have liquefied to their total depth and are at their assumed residual shear strength (USR=0.05), then calculate the static factor of safety (FoS) and yield acceleration. For the current design concept to have credibility for advancing it to more detailed design, a calculated static FoS must exceed 1.0 and the yield acceleration (ky) must exceed ½ the peak ground acceleration (PGA) of the design earthquakes.**

It is unclear the reference or source indicating the requirement that the yield acceleration (ky) must exceed ½ the PGA of the design earthquake. The design of the embankment structures, including detailed stability analyses will be completed as per the design requirements outlined in the ADSP.

The upstream static FoS for a fully liquefied tailings mass is presented above.

Downstream static stability analysis considering fully liquified tailings (shear strength of 0.05) are present in Figure 4 below, with a resulting FoS above 1.8. The liquefaction of the tailings does not affect the downstream stability of the embankment.

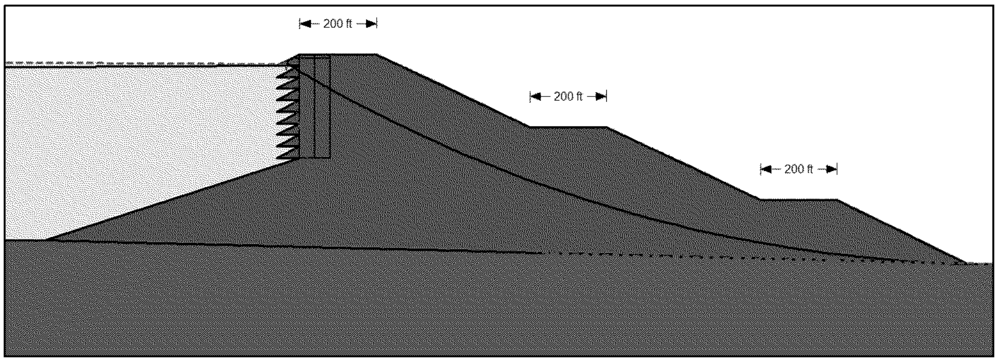

Figure 4 – Downstream Static Stability – Post Liquefaction

The yield acceleration of the Main Embankment was determined to be 0.32 g using the computer modelling software Slope /W where the yield acceleration corresponds to a factor of safety of 1.0. Table 3 provides a summary of the ky/PGA values for the four design earthquakes.

Table 3 ky/PGA

Event	Peak Ground Acceleration	ky/PGA
Magnitude 9.2 interface subduction earthquake associated with the Alaska-Aleutian Megathrust	0.16g	2.0
Magnitude 8.0 deep intraslab (in-slab) subduction earthquake	0.61g	0.53
Magnitude 7.5 shallow crustal earthquake on the Lake Clark fault (Mapped)	0.32g	1.0
Magnitude 6.5 maximum background earthquake (shallow crustal event assumed to occur directly beneath potential mine site facilities)	0.56g	0.57

d. On the basis of the above items #7a, b, and c, the potential liquefaction of the deeper tailings under and adjacent to the footprint of the upstream parts of the centerline raises is a concern with respect to the stability of the upstream slopes, and potentially of the raises themselves. A concern is that deep tailings could suddenly liquefy under static or dynamic (earthquake) loading, causing a containment failure and release that cannot be practically mitigated in a timely way. The RFI 008 series documents reduce but do not eliminate that risk. Therefore, given what is known and not known, it may be not be reasonable to preclude nor even quantify that potential. The practical bottom line is that the risk is undefined, but high. Please describe how the embankment planning, design, construction, and operations would be conducted to reduce this risk.

The upstream stability of the Main Embankment under post liquefaction conditions was addressed in KP 2019b and above in the response to question 7b. The analyses conclude that “Liquefaction of the tailings mass could result in some deformation of the upstream Zone U material, particularity the upstream edge of the

	material that has been constructed onto the drained tailings beach. The deformations are expected to be constrained within the upstream zone of the dam with no loss of freeboard or compromise to the integrity of the structure (KP, 2019b)". The minimum design FoS for post-earthquake, or post-liquefaction, conditions is 1.2. Liquefaction of the tailings mass will not result in a containment failure. Development of the embankment design, construction, and management during operations will be completed based on the ADSP program guidelines for a Class 1 embankment. Ongoing evaluations of the design criteria and concepts will be completed throughout the preliminary and detailed design phase and will be updated based on information gathered during future studies. The OMS manual will outline several maintenance and monitoring requirements for the facility and will be continually updated as required throughout operations and closer. **8) Also at this point of the design process, it is prudent to conduct sensitivity analyses on the yield acceleration (ky) criterion. For example, ky could be reduced to ¼ PGA or arguably even a bit lower (increasing potential deformation). The ratio of ky/PGA can be considered an index of potential embankment displacement, with the potential displacement increasing with decreasing ky/PGA and vice versa. There is no fundamentally correct criterion. A ky/PGA=0.50 could be considered as "too conservative," allowing "too little displacement," so not acceptable. A lower value, like ky/PGA=0.25, would be less open to being considered "too conservative to be acceptable." Please conduct sensitivity analyses to compare calculated displacements as a function of ky/PGA before suggesting an acceptable value.** The detailed design of the embankment structures will not rely on a simplified deformation analysis that considers only the yield acceleration (Ky). Preliminary stability analyses under the four design earthquake events result in a ky/PGA of greater than 0.5 as presented above. A more detailed assessment is required given the very different ground motion characteristics (defined by the earthquake magnitude, frequency content and amplitude) associated with the four MCE scenarios defined for dam design. Consequently, we have conducted preliminary semi-empirical seismic deformation analyses using the Bray method. This is a more rigorous assessment that considers not only the yield acceleration but also the design earthquake magnitude, peak ground acceleration, the fundamental period of the dam (defined by material stiffness and dam height) and the corresponding spectral acceleration provided by the design response spectrum. Numerical modelling will be completed as part of the design phase to estimate potential displacements within the structures. **9) Case 3 in KP (2019b) assumes that the phreatic surface would be held at or near the bottom of the main embankment, and that there would be no earthquake-induced pore pressures in embankment materials. However, it is possible that the embankment bulk, filter, and transition zone materials could impede the flow-through concept because of incompatible gradations, particle deterioration, filter failure, chemical precipitation, etc., thereby causing the phreatic surface in the embankment to rise and the FoS to decrease. Please provide:** **a. Backup documentation that supports the assumption that the phreatic surface would be near the bottom of the embankment and stay there for the duration of the TSF operations and into closure and post-closure;** The Bulk TSF Main embankment is proposed as a zoned, earthfill/rockfill embankment to be constructed with a number of engineered zones including filter

and transition zones. Durability testwork will be completed on the filter and transition zone materials to confirm they are suitable for use in the dams. The engineered materials zones (including the rockfill zone) will control migration of materials between adjacent fill zones, and provide the drainage capacity within the embankment structure. This normal operating condition was applied to the preliminary analysis competed to date.

Detailed seepage modelling will be completed as part of the ADSP to model and confirm the preliminary phreatic surface assumed in these analyses.

b. A discussion of the confidence level in the rockfill gradations and quality expected to be produced from on-site quarries and the ability to maintain these gradations and resist particle degradation; and

The engineered materials processed on site will be subject to ongoing quality control and assurance testing, which will be consistent with industry best practice and defined in the design documentation. Testing will include Particle Size Distribution, durability testing, ARD tests, and moisture control depending on the material and application. The frequency of the testing (based on volume or tonnage) will be defined during the detailed design phase of the ASDP. The quality control systems will provide confidence in the ongoing quality of the materials. Material that does not meet the required specification will not be used in embankment construction.

c. Conduct sensitivity analyses that assume a much higher phreatic surface in the embankment.

The downstream static stability of the post-liquefaction condition (tailings shear strength of 0.05) was assessed using a revised phreatic surface, which is considerably higher than that applied for normal operating conditions, as shown on Figure 5. This model assumes the phreatic surface remains just below the crest of the embankment for half the width of the embankment crest. This modelled scenario assumes the filter and transition zones are fully blocked and act similar to a core zone within the embankment, with the downstream rockfill shell providing drainage capacity. This scenario is highly unlikely considering the typical characteristics of rockfill material.

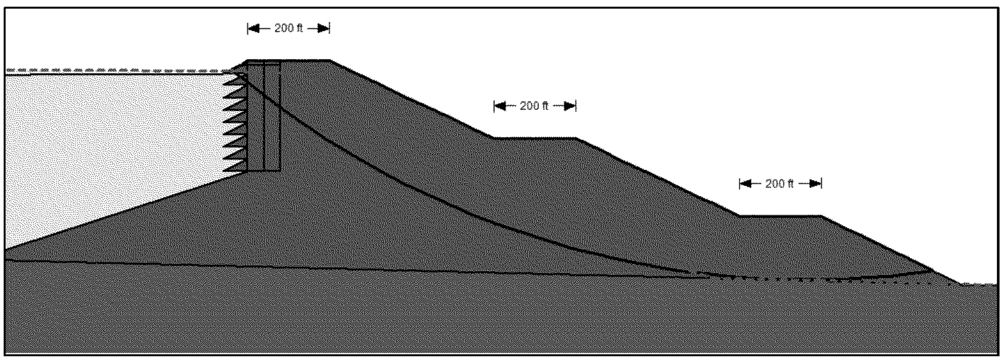

Figure 5 - Downstream Stability – Post Liquefaction, Higher Phreatic Surface

The resulting FoS with the higher phreatic surface is in the range of 1.8 to 2, slightly lower than the previous static stability results. The stability analysis will be updated on an ongoing basis as the preliminary and detailed design phases of the ADSP are advanced.

	10) The initial RFI 008g response refers to allowable and calculated FoS in several places. As is known in the geotechnical engineering practice and described in Alaska Dam Safety Program guidelines (ADNR 2017), a calculated FoS is only as reliable as the quality of the data that the calculation is based on and the level of engineering analyses completed. Please discuss the confidence levels of each of the allowable and calculated FoS in RFI 008g with respect to the confidence of their underlying assumptions, level of engineering analyses completed, and published data versus site-specific data. The preliminary stability analysis have been completed using material and foundation parameters based on available site specific data and rely on multiple sources of engineering literature as previous outlined. The preliminary analysis also assumed a single homogeneous rockfill and single unit foundation conditions. The analysis completed to date is considered preliminary, however is based on sound engineering judgement considering the level of the current design. As part of the preliminary and detailed design phases of the ADSP, a detailed stability analysis will be completed and updated as required based on: • Foundation conditions determined during previously completed and planned site investigation programs. • Material parameters and the inclusion of internal embankment zones, • Detailed seepage analysis and refinement of the phreatic surface within the embankment materials, and • Tailings parameters determined from site specific testwork. **11) Provide examples of similar size and shape tailings embankments built by centerline construction methods worldwide where they have worked successfully before and are still working successfully. Describe operational issues at similar facilities related to tailings liquefaction potential and embankment stability, and how they were addressed.** See attached Table 1 for a list of comparable centerline tailings dams. Operational issues related to liquefaction and embankment stability are not included as limited public information is available. The preliminary stability analysis for the Bulk TSF Main embankment considering fully liquefied tailings is provided in the response to question 7, and response to question 5 provides information on the operational actions that may be implemented if increased pore pressures are identified within the tailings. **12) A main part the NEPA analysis is to compare alternatives. Please discuss the impacts that deep tailings liquefaction in the bulk TSF would have on the main embankment and on the TSF in general if the main embankment was built by downstream construction methods to its full ultimate height.** Full tailings liquefaction of a downstream main embankment alternative was not modelled, however it can be expected the downstream alternative will result in a similar FoS with minimal settlement along the upstream crest. The tailings materials will undergo similar post liquefaction consolidation and shear strength reduction regardless of the embankment construction method (centerline vs downstream). **<u>References</u>** Abrahamson, N.A., Gregor, N. and Addo, K, 2016, “BC Hydro Ground Motion Prediction Equations for Subduction Earthquakes”, Earthquake Spectra 32, p. 23-44.

	KP. 2019a. Report on Seismicity Assessment and Seismic Design Parameters. Prepared for PLP, VA101-176/60-1, July 4, 2019. KP. 2019b. Memorandum to S. Hodgson (PLP), Re: Main Embankment Stability Assessment – Static and Post-liquefaction. Cont. No. VA19-00587, July 8, 2019. Newmark, N.M. 1965. Effects of earthquakes on dams and embankments. Géotechnique, 15(2): 139-160. Petersen, M.D. et al, 2018, Preliminary 2018 Update of the U.S. National Seismic Hazard Model: Overview of Model, Changes, and Implications. Vick, S.G., 1990. Planning Design and Analysis of Tailings Dams. John Wiley & Sons. Zhao, J.X. et al, 2006, “Attenuation Relations of Strong Ground Motion in Japan Using Site Classification Based on Predominant Period”, Bulletin of the Seismological Society of America, Vol. 96, No. 3, p.898-913.
List Number and Type of Response Attachments:	**T1 Comparison of Centerline Constructed Dams.pdf**
Date Returned to USACE:	Click here to enter text.

AECOM Intake Form

Date Response was Received:	9/20/2019
Received by:	AECOM
Describe any Follow-up Related to this RFI:	Click here to enter text.

Knight Piésold CONSULTING

TABLE 1

PEBBLE LIMITED PARTNERSHIP
PEBBLE PROJECT

RESPONSE TO REQUEST FOR INFORMATION 008h
SUMMARY OF COMPARABLE CENTERLINE DAMS

Print Sep/20/19 8:29:02

Site	Location	Years of Operation	Dam Height	Construction Method	Embankment Construction Material	References
Alumbrera	Argentina	1997 to Present	540 ft (Projected)	Modified Centerline	Rockfill / Earthfill	1. Kostaschuk,R., Brouwer, K., and J. Haile. 2000. Continuity is the key. Water Power and Dam Construction. October 2000. https://www.waterpowermagazine.com/features/featurecontinuity-is-the-key/
Brenda	British Columbia, Canada	1970 to 1990	450 ft	Centerline	Rockfill starter dam with cycl one sand shells.	1. Klohn, E.J. 1984. The Brenda Mines' Cyclone-Sand Tailings Dam. International Conference on Case Histories in Geotechnical Engineering. pg 953 to 977.
Cerro Verde	Peru	2006 to Present	Permitted to 985 ft	Centerline	Cyclone sand dam with a zoned rockfill starter dam.	Obermeyer, J., Alexieva, T. 2011. Design, Construction and Operation of a Large Centerline Tailings Storage Facility with High Rate of Rise. Vancouver, B.C. Proceedings Tailings and Mine Waste 2011.
Constancia	Peru	2014 to Present	>328 ft	Centerline	Zoned rockfill with a vertical clay core z one.	1. Ridlen, P.W., Kerr, T.F., Domınquez, G. Varnier, J.B. 2018. Design of a Centerline Method Tailings Dam using Mine Waste Rockfill in Peru. Tailings and Mine Waste 2018.
Gibraltar	British Columbia, Canada	1972 to Present	385 ft	Centerline	Cyclone Sand with rock core	1. KCB. 2014. Gibraltar Mine Tailings Storage Facility 2014 Annual Dam Safety Inspection. November 26. Vancouver, B.C. Ref No. M01527A75.730. 2. KCB. 2018. Gibraltar Mine Tailings Storage Facility 2017 Annual Dam Safety Inspection. March 2018. Vancouver, B.C. Ref No. M01527A89.730.
Highland Valley Copper - Highland TSF	British Columbia, Canada	1972 to present	L-L Dam: 528 ft H-H Dam: 318 ft	Centerline	L-L Dam: earthfill starter dam with a low permeability vertical core, with an upstream cyclone sand berm/tailings beach and cyclone sand downstream. H-H Dam: earthfill dam with a low perm eability vertical core, with random fill and tailings placed upstream and variable waste fill on the downstream side.	1. KCB. 2018. 2017 Dam Safety Inspection Report Highland Tailings Storage Facility. March 29. Vancouver, B.C. Ref No. M02341B26.730. 2. Scott, M.D., Klohn, E.J., Lo, R.C., Lum, Ken.K. 1988. Overview of Highland Valley Tailings Storage Facility. International Conference on Care Histories in Geotechnical Engineering. 34.
Fort Knox	Alaska, USA	1996 to Present	350 ft	Downstream to Modified Centerline	Rockfill	1. Kinross Gold Corporation, Fort Knox mine, Fairbanks North Star Borough, Alaska, USA, NI 43-101 Technical Report
Montana Resources	Montana, USA.	1963 to Present	750 ft	Centerline	Rockfill	1. Hydrometrics Inc. 2018. Baseline Hydrology Report for the Yankee Doodle Tailings Impoundment Amendment to Operating Permits 00030 and 00030A. January. Helena MT. 2. Montana Resources. 2018. Amendment to Operating Permits 00030 and 00030A to Continue Operations at the Continental Mine. January. Butte, MT.
Montana Tunnels	Montana, USA.	1987 to Present	Permitted to 410 ft in 2008.	Modified Centerline	Rockfill	1. Haile, J.P., Brouwer, K.J. 1994. Modified Centerline Construction of Tailings Embankments. 3rd International Conference on Environmental Issues and Waste Management in Energy and Mineral Production, August. Perth Australia. 2. State of Montana Department of Environmental Quality, United States Department of the Interior Bureau of Land Management. 2008. Final Environmental Impact Statement FES 08-31 for the Proposed M-Pit Mine Expansion at the Montana Tunnels Mine in Jefferson County, Montana.
Thompson Creek Mine - Bruno Creek TSF	Idaho, USA	1983 to 2014 (Care and Maintenance: 2014 to Present)	558 ft	Centerline	Cyclone sand dam with earthen starter dam.	1. U.S. Environmental Protection Agency Office of Solid Waste. 1992. Mine Site Visit: Cyprus Thompson Creek. June. Washington D.C.

\\KPL\VA-Prj$\1\01\00176\60\A\Data\Task 300 Response to RFIs\RFI 008 Stability\RFI 008h\3. Dam Comparison\[T1 Comparison of Ce nterline Constructed Dams.xlsm]Table 1

REV	DATE	DESCRIPTION	PREP'D	RVW'D
0	20SEP'19	ISSUED WITH TRANSMITTAL VA19-001709	JRG	LJG